Research Report
394

SHEAR STRENGTH OF COHESIVE SOILS AND FRICTION SLEEVE RESISTANCE

by

V. P. Drnevich
Associate Professor of Civil Engineering
University of Kentucky

C. T. Gorman
Research Engineer Assistant

and

T. C. Hopkins
Research Engineer Principal

Division of Research
Bureau of Highways
DEPARTMENT OF TRANSPORTATION
Commonwealth of Kentucky

prepared for presentation to
European Symposium on Penetration Testing
Stockholm
June 5-7, 1974

June 1974

SUMMARY

Cone penetrations tests were performed on the silty clays of Kentucky, U.S.A., using a boring rig to push the Dutch, friction sleeve, cone penetrometer. Thin-walled tube samples were taken from nearby boreholes. For the first four sites, unconfined compression tests and unconsolidated-undrained triaxial tests were performed on the samples. For the last four sites, consolidated-undrained triaxial tests were performed on the samples. A procedure for estimating in situ shear strength from triaxial test stress paths was developed.

Small rock fragments in these residual soils caused erratic cone resistance at many locations. As a result, the friction sleeve resistance provided the best correlation with in situ shear strength. In situ shear strength was found to be approximately 80 percent of the friction sleeve resistance, which confirms the findings of others.

INTRODUCTION

Dutch cone penetration testing was initiated at the University of Kentucky, USA, in 1971. Early efforts by Cleveland (1971) focused on the correlation of Dutch cone penetration test results with standard penetration test, soil type identification, laboratory vane shear test, unconfined compression test, and unconsolidated undrained triaxial shear test results. Cleveland's findings indicated that a relationship existed between Dutch cone friction sleeve resistance (Begemann, 1953) and shear strength, as measured by unconsolidated undrained triaxial tests.

In September 1972, a cooperative effort between the Kentucky Department of Transportation and the University of Kentucky was initiated to further assess the capabilities of the Dutch cone penetration test as a means of determining in situ shear strength. Several highway landslide sites were chosen for investigation. This venture provided the opportunity to expand shear strength correlations to a wide variety of soils. Among the soils tested were compacted embankments, residual silty clays, and alluvial deposits of a more silty nature. Conditions of full and partial saturation and normal and over consolidation existed. The results of both studies are presented herein.

BACKGROUND

The first attempts at predicting shear strength using the Dutch cone penetrometer involved the correlation of cone resistance, q_c, with shear strength. From bearing capacity theory and equations, an equation relating undrained shear strength, τ, to cone resistance, q_c, an empirical bearing capacity factor, N_c, and overburden pressure, P_o, may be derived (Thomas, 1965). This equation is of the form

$$\tau = (q_c - P_o)/N_c.$$

However, P_o may be neglected, yielding the equation

$$\tau = q_c/N_c.$$

Research correlating q_c with undrained shear strength, as determined by various methods, has yielded values of N_c ranging from 5 to 25 (Sanglerat, 1972).

Development of the friction sleeve by Begemann (1953) offered another approach to the determination of undrained shear strength. Begemann (1965) suggested that the value of friction resistance, f_s, should be approximately equal to the undrained shear strength. This view was supported by Tomlinson's (1957) work with piles in clay soils. Tomlinson found that pile adhesion was approximately equal to soil cohesion, C_u, for soft clays. Similar research by Vesic (1969) limited this relationship to soils with undrained shear strengths less than 0.7 kg/cm^2. Experimental correlation of the relation between Dutch cone sleeve friction and undrained shear strength was presented by Wesley (1967) and showed sleeve friction to be slightly higher than undrained shear strength.

TESTING PROCEDURE

The Dutch friction cone penetrometer was adapted to a conventional boring rig as described by Drnevich (1974). Dutch cone penetration testing was performed at four highway landslide sites in this study. These sites offered the opportunity to investigate both compacted embankments and foundation soils. Conditions of both full and partial saturation existed, and rock fragments were encountered in a few cases. Penetration test results for the four sites are shown in Figure 1. (Cleveland's work was performed at naturally occurring

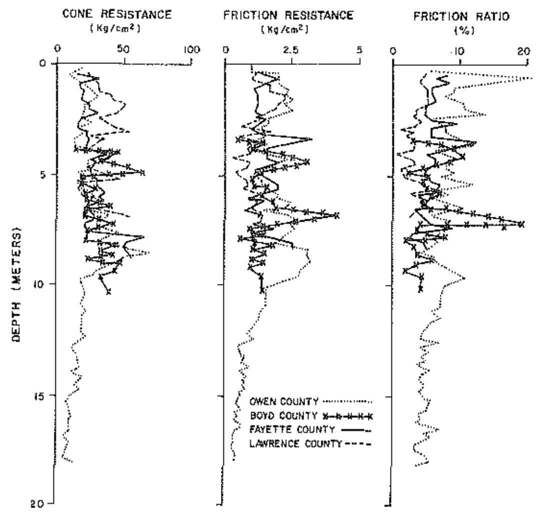

Figure 1. Dutch Cone Penetration Test Results

deposits of residual silty clays and alluvial clayey silts.) Thin wall Shelby tube samples were taken near the Dutch cone penetration test holes. These "undisturbed" samples were used in subsequent triaxial testing to determine "in situ" shear strength. However, in the sampling process the in situ total stresses are removed from the sample and some disturbance is inevitable. To overcome this problem, initial in situ conditions were duplicated for one sample of each set of triaxial tests by consolidating it to the mean in situ effective stress, σ_e', given by the equation:

$$\sigma_e' = 1/3\ (1 + 2K_o)\ \sigma_v'.$$

K_o varies with soil origin, soil type, and load history. For a given soil deposit, K_o varies with the degree of overconsolidation, which may be affected by dessication near the surface, water table fluctuations, and sedimentation and erosion. Test results published by Bishop and Henkel (1957) for compacted embankment soils show values of K_o ranging from 0.35 to 0.65. The lower values of K_o pertain to soils having a low percentage of clay fraction. Generally the Dutch cone tests were performed in soils having a high percentage of clay fraction. Hence the K_o values for the compacted embankment soils could be expected to tend toward the higher range of the K_o values. An estimate of K_o for the foundation soils encountered was made using test results published by Lambe and Whitman (1969) which gives K_o as a function of overconsolidation ratio and plasticity index. The range of plasticity index (0-20) and overconsolidation ratio (1-4) encountered in these soils yield a range of K_o from 0.40 to 0.80. K_o was assumed to be 0.62 for both cases as this value tended toward the higher range for compacted fills and was a median value for the foundation soils. Substituting this value into Equation 1 yields

$$\sigma'_c = 3\ \sigma'/4$$

Following isotropic consolidation of the laboratory specimen to σ'_c, the drainage lines were closed and the sample loaded axially, thereby reproducing undrained failure.

An example of a plot of triaxial test data is shown in Figure 2. Note that the stress path method (Simons, 1960; Lambe, 1964) is used to show the continuous stress change during loading. Tests under in situ conditions were used to determine the shear stress on the failure plane, τ_f. Assuming in situ failure stresses are mobilized when the in situ stress path intersects the K_f line, the Mohr circle at failure can be defined from the point of intersection. The values of τ_f may be determined from q_f and ϕ'. Derivation of the equation

$$\tau_f = q_f \cos \phi'$$

is shown in Figure 3.

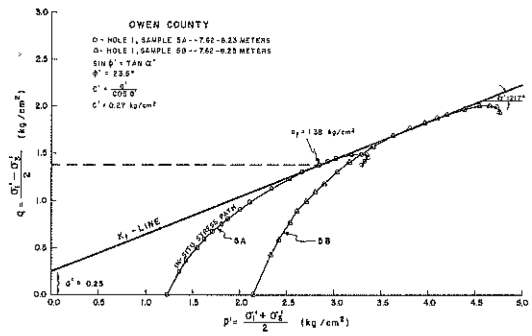

Figure 2. Typical Triaxial Test Data

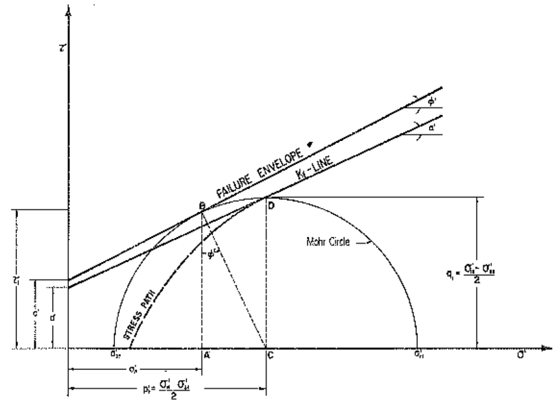

Figure 3. Derivation of the Equation $\tau_f = q_f \cos \phi$

RESULTS

Index properites of the soils encountered are shown in Table 1. Results of Dutch cone penetration testing and triaxial testing performed on undisturbed samples from these sites are summarized in Table 2. A statistical analysis of the data produced a regression line with the equation $f_s = 1.28\ \tau_f$ to describe the data.

Work done by Cleveland yielded similar results. When subjected to the same statistical analysis, Cleveland's data resulted in an equation

Table 1. Index Properties of Soils at the Test Sites

LOCATION	DEPTH (METERS)	WATER CONTENTS (PERCENT)				LIQUIDITY INDEX	UNIFIED CLASSIFICATION	GRADATION (PERCENT)		
		NATURAL	LIQUID LIMIT	PLASTICITY INDEX	DEGREE OF SATURATION			SAND > 0.074 MM	SILT 0.074 - 0.0005 MM	CLAY < 0.005 MM
OWEN CO.	0.0 - 12.2	20	40	19	95 - 100	-0.05	CL	4	46	50
OWEN CO.	12.2 - 18.3	27	37	16	100	0.38	CL	3	56	41
FAYETTE CO.	0.0 - 9.1	24	36	18	95 - 100	0.33	CL	20	39	41
BOYD CO.	0.0 - 13.7	11	35	14	50 - 100	-0.71	CL	19	41	40
LAWRENCE CO. BOREHOLE 11A	4.6 - 7.6	20	21	1	100	0.00	SM	60	26	14
LAWRENCE CO. BOREHOLE 8A	0.0 - 6.1	21	24	4	98 - 100	0.25	ML - CL	45	36	19
UNIV. OF KY.* CAMPUS	0.0 - 3.1	30	39	17		0.47	CL	17	30	53
UNIV. OF KY.* POULTRY FARM	0.0 - 3.1	35	56	24		0.12	MH - CH	46	11	43
KENTUCKY RIVER* LOCK NO. 9	0.0 - 9.4	22	28	8		0.25	CL - ML	22	48	30
LEXINGTON, KY.* POST OFFICE	0.0 - 3.1	27	43	14		-0.14	CL - ML	12	40	48

*BY CLEVELAND

Table 2. Summary of Triaxial and Dutch Cone Data

SITE	BOREHOLE NUMBER	DEPTH (METERS)	TRIAXIAL DATA			DUTCH CONE DATA			
			SHEAR STRENGTH PARAMETERS ϕ' (DEGREES)	SHEAR STRENGTH PARAMETERS c' (kg/cm²)	UNDRAINED SHEAR STRENGTH τ_f (kg/cm²)	SOUNDING NUMBER	LOCATION	FRICTION SLEEVE RESISTANCE f_s (kg/cm²)	CONE RESISTANCE q_c (kg/cm²)
OWEN CO.	1	4.6 - 5.2	27.6	0.41	1.08	1	1.5 m from BH 1	1.73	22.7
		6.1 - 6.7							
		7.6 - 8.2	27.0	0.092	1.27			2.23	42.0
		9.1 - 9.8	30.8	0.00	0.97			2.58	26.7
		10.7 - 11.3	19.8	0.49	1.18			1.38	20.3
		13.7 - 14.3	31.7	0.04	0.51			0.38	12.5
		16.8 - 17.4							
		15.2 - 15.8	31.5	0.09	0.95			0.40	9.7
		16.8 - 17.4							
FAYETTE CO.	1	4.6 - 5.2	21.3	0.57	1.03	3	0.9 m W of BH 1	1.31	25.7
		6.1 - 6.7	29.6	0.07	0.84			1.11	26.7
		7.6 - 8.2	19.1	0.58	1.11			1.69	49.3
	2	2.4 - 3.0	25.5	0.38	0.89	4, 5, 6	2.3 m W, 1.2 m W, 0.9 m E of BH 2.	1.20	26.9
		4.6 - 5.2							
BOYD CO.	1	3.0 - 3.7	22.6	0.60	1.45	3	20.6 m E of BH 1	1.18	29.5
		4.6 - 5.2							
		6.1 - 6.7	21.4	0.66	1.46			1.62	31.9
		7.6 - 8.2							
	2	7.6 - 8.2	30.6	0.006	0.99	1	4.7 m W of BH 2	1.57	27.5
		10.7 - 11.3	21.2	0.53	1.60			2.00	41.0
LAWRENCE CO.	8A	1.5 - 2.1	33.0	0.12	0.68	8	1.5 m S of BH 8A	0.91	19.3
	8	3.7 - 4.3	27.4	0.34	1.12		1.5 m S of BH 8A	0.91	39.0
	8A	3.0 - 3.7					3.7 m SE of BH 8		
	11	5.2 - 5.5					24.9 m S of BH 11		
		6.4 - 7.0	27.8	1.53	0.72	11	& 1.5 m S of BH 11A	0.90	33.2
	11A	4.6 - 5.2							

of $f_s = 1.19\ \tau_f$. However, Cleveland reproduced in situ conditions in an unconsolidated, undrained triaxial test by applying stresses equal to the full overburden pressure to the sample. In this research, in situ conditions were reproduced in a consolidated, undrained triaxial test by applying effective stresses equal to 3/4 of the overburden pressure. Combining data from this research with Cleveland's data resulted in a regression equation of $f_s = 1.24\ \tau_f$ (see Figure 4).

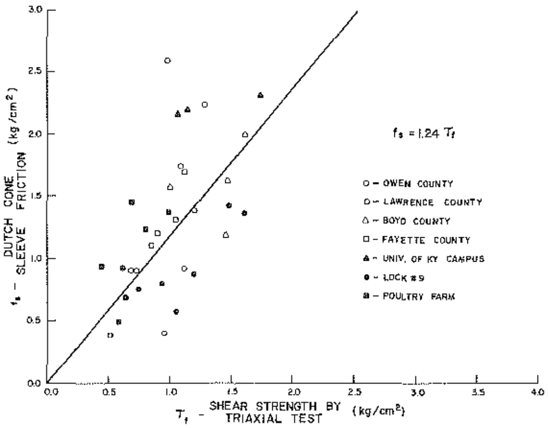

Figure 4. Relationship between Dutch Cone Sleeve Friction and Undrained Shear Strength

DISCUSSION

The results of this study and the results of Cleveland (1971) and Wesley (1967) show very close agreement. Shown in Figure 5 are the relations between friction sleeve resistance, f_s, and undrained shear strength resulting from the three independent studies. In all cases, f_s was found to be slightly higher than the undrained strength as measured by laboratory tests. Begemann initially set undrained shear strength as the upper limit for sleeve friction; however, Wesley attributed the higher values of f_s to secondary loads (forces acting on the bevelled lower edge of the friction sleeve) and high penetration rate.

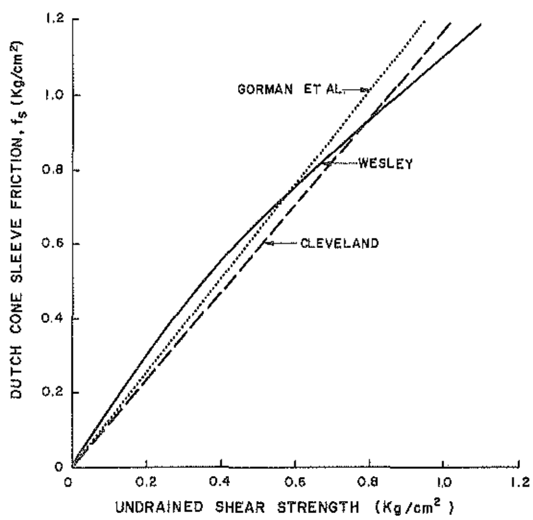

Figure 5. Comparison of Various Relationships Between Dutch Cone Sleeve Friction and Undrained Shear Strength

The difference in the mechanisms of failure should be considered in any discussion of shear strength and Dutch cone sleeve friction. In the triaxial test, or in situ, undrained shear strength is the shear stress on a soil-soil interface known as the failure plane. This plane forms an oblique angle with the vertical which is usually unknown. Dutch cone sleeve friction, however, is the frictional resistance developed along a vertical steel-soil interface. This difference makes theoretical correlation of the two quantities extremely difficult. Therefore, empirical correlation seems to offer the best means of associating the two quantities.

No corrections were applied to the Dutch cone sleeve friction values to account for the differences in soil type or conditions. Thus the correlations shown in Figure 4 represent a wide variety of soil types and conditions of saturation and consolidation.

In addition, experimental scatter may be expected in both triaxial and Dutch cone testing. Triaxial test scatter can be caused by disturbances during sampling and trimming of the specimen and vertical variation in the soils tested for a given set of triaxial data. In situ conditions were "duplicated" in the triaxial test by isotropic consolidation of the specimen using a value of K_0 equal to 0.62. Lateral in situ stresses are difficult, at best, to predict and most certainly varied for the soils tested.

Dutch cone soundings were taken at various distances from the bore holes from which the undisturbed samples were taken. Any lateral variation in soil properties could also lead to variations in shear strengths, which in turn could produce scatter unrelated to the test methods.

CONCLUSIONS

For a variety of cohesive soils that include residual silty clays, compacted embankments, and alluvial clayey silts, undrained shear strength as measured by triaxial tests was found to be approximately 80 percent of the friction sleeve resistance as measured by the Begemann friction sleeve cone penetrometer. Friction sleeve resistance provided a better correlation with undrained shear strength than did cone resistance. This could be due in part to encountered rock fragments having less an effect on the friction sleeve resistance than on the cone penetration resistance.

Unconsolidated-undrained and consolidated-undrained triaxial tests were performed. In the former, the confining pressure was equal to the total overburden stress, and in the latter, the effective confining pressure was made equal to 75 percent of the mean effective principal stress. Both types of tests yielded approximately the same correlation, implying that the unconsolidated-undrained type of test is sufficient.

Thus, a rough estimate of undrained shear strength may be obtained from friction sleeve resistance using the correlation developed herein. For more accurate determinations of in situ shear strength, it is recommended that correlations be established at a given site.

REFERENCES

BEGEMANN, H. K. S., 1953. Improved Method of Determining Resistance to Adhesion by Sounding through a Loose Sleeve Placed behind the Cone. Proceedings, International Conference on Soil Mechanics and Foundation Engineering, Zurich, Vol 1, pp 213-217.

BEGEMANN, H. K. S., 1965. The Friction Jacket Cone as an Aid in Determining the Soil Profile. Proceedings, International Conference on Soil Mechanics and Foundation Engineering, Montreal, Vol 1, pp 17-20.

BISHOP, A. W. and HENKEL, D. J., 1957. The Measurement of Soil Properties in the Triaxial Test. Edward Arnold Publishers, Ltd., London, Table 2, p 143.

CLEVELAND, E. P., 1971. Use of the Dutch Cone Penetration Test for Soil Exploration in Kentucky. Thesis presented to the University of Kentucky in partial fulfillment of the requirements for the degree of Master of Science.

DRENEVICH, V. P., 1974. Use of Conventional Boring Rigs for Penetration Testing. Proceedings, European Symposium on Penetration Testing, Stockholm, Vol 2.
LAMBE, W. T., 1964. Methods of Estimating Settlement. Journal of the Soil Mechanics and Foundations Division, ASCE, Vol 90, No. SM5, pp 43-67.
LAMBE, W. T. and WHITMAN, R. V., 1969. Soil Mechanics. John Wiley and Sons, Inc., New York, Figure 20, p 300.
SANGLERAT, G., 1972. The Penetrometer and Soil Exploration. American Elsevier Publishing Co., Inc., New York.
SIMONS, N. W., 1960. The Effect of Overconsolidation on the Shear Strength Characteristics of an Undisturbed Oslo Clay. Proceedings, Research Conference on Shear Strength of Cohesive Soils, ASCE, June 1960.
THOMAS, D., 1965. Static Penetration Tests in London Clay. Geotechnique, Vol 15, No. 2, pp 174-179.
TOMLINSON, M. J., 1957. The Adhesion of Piles Driven in Clay Soils. Proceedings, International Conference on Soil Mechanics and Foundation Engineering, London, Vol 2, pp 66-71.
VESIC, A. B., 1969. Discussion, Proceedings, International Conference on Soil Mechanics and Foundation Engineering, Mexico, Vol 3, pp 242-244.
WESLEY, L. D., 1967. The Dutch Penetrometer and Its Use in Indonesia. Proceedings, Southeast Asian Regional Conference of Soil Engineering, Bangkok, pp 223-230.